ON INELASTIC RESPONSE SPECTRA FOR ASEISMIC BUILDING DESIGN

by

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SUMMARY

The variability of inelastic response spectra is investigated by time-history analysis using sets of artificial ground motions with different strong-motion durations. It is concluded that the Newmark-Hall inelastic response spectra of elastoplastic systems are unconservative for 5% damping, and conservative for 2% damping. New inelastic response spectra are proposed for different ductility ratios and damping coefficients. A 10-story steel moment-resisting frame is designed and analyzed to assess the validity of the design procedure based on modal analysis and inelastic response spectra. The results indicate that the method is generally satisfactory, except that it would lead to slightly unconservative design for upper-story exterior columns.

INTRODUCTION

The use of a "response spectrum" to characterize structural response has been well established in aseismic engineering. Newmark and Hall (2) suggested an approximate procedure to construct the inelastic relative displacement response spectrum (IDRS) and the inelastic absolute acceleration response spectrum (IARS). As illustrated in Fig. 1, for intermediate and high natural period ranges, the IARS equals the elastic response spectrum divided by the ductility ratio μ, whereas the IDRS is the same as the elastic spectrum. In the intermediate low natural period range, the IARS equals the elastic spectrum divided by $(2\mu-1)^{1/2}$, which is derived from the principle of energy conservation. In the same range, the IDRS equals the elastic spectrum times $\mu/(2\mu-1)^{1/2}$. For the very low natural period range, the system is very stiff, and the IARS equals the elastic one, whereas the IDRS equals the elastic spectrum times μ.

VARIABILITY OF THE INELASTIC RESPONSE SPECTRA

In order to investigate the variability of inelastic response spectra for variations in strong-motion duration, ductility ratio, damping ratio, etc., four sets of artificial motions with durations of 10, 20, 30, and 40 seconds have been generated by matching the Newmark-Hall elastic design response spectrum. Each set consists of five different motions with a peak acceleration of 1.0 g. In this study, 50 single-degree-of-freedom systems with natural periods equally spaced between 0.1 sec and 10 secs on the logarithmic scale are analyzed. By varying the resistance function for each of these SDOF systems, the elasto-plastic responses for a given ductility ratio can be calculated by time-history analysis. The mean E-P response of each SDOF system can then be computed for each set of artificial motions with varying strong-motion duration.

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Fig. 1 shows typical results of the mean values for IARS and IDRS for different ground-motion durations (μ=5, 5% damping). For each of the 50 natural periods considered the mean IARS or IDRS does vary with different motion durations. However, there is no unique trend with respect to the variations of duration. Hence, it can be concluded that, for a given ductility ratio, the inelastic responses are not significantly dependent on strong-motion duration. The mean inelastic responses shown in the figure also suggest that the Newmark-Hall procedure of constructing inelastic response spectra is unconservative for 5% damped E-P systems. The unconservatism increases with the ductility ratio. The same computation procedure has been applied to 2% damped E-P systems. Contrary to the results of 5% damped systems, the mean values of IARS and IDRS for 2% damped systems are generally somewhat smaller than those of the Newmark-Hall approach. This suggests that the Newmark-Hall procedure predicts slightly conservative response for 2% damped E-P systems. The conservatism increases with decreasing ductility ratio. The above-mentioned conclusions are quite consistent with the results of a recent study by Riddell and Newmark (3).

Based on the results discussed above, new inelastic response spectra are proposed. For example, Fig. 2 shows the resulting IARS and IDRS for μ=4 and 5% damping with 1.0 g peak ground acceleration. These new spectra are employed in this study to investigate the reliability of a design procedure based on inelastic response spectra and modal analysis.

FRAME DESIGN PROCEDURE USING INELASTIC RESPONSE SPECTRA

Given the member sizes determined in a preliminary design, and the design ductility ratio of the structure, as well as the design peak ground acceleration, the maximum member forces can be computed by modal analysis using the scaled IARS. From the results, the structure is designed to provide the required strength in each member. The SRSS modal superposition method is used in this study. Based on the argument that static end moments do not alter the member plastic capacity after its first yield, the required girder moment capacity, M_y, is determined as the maximum value of M_{EQ} or $w\ell^2/8$. M_{EQ} is the average of the girder end moments computed by the modal analysis using IARS. The second criterion, $w\ell^2/8$, is to ensure against the undesirable formation of a plastic hinge at girder midspan due to the uniform gravity load, w.

Column moment capacity is determined by the AISC axial-flexural interaction formula (Sec. 2.4-3, 1973): $(P/P_y) + (M/1.18\ M_y) \leq 1.0$, and $M_y \geq M$. With the assumption that the ratio of plastic modulus Z to area A is approximately equal to 6 for column sections of interest, the interaction formula becomes: $M_y \geq 6P+M/1.18$, $M_y \geq M$, and $P_y = M_y/6$, whereas $P = P_{EQ} + P_{GR}$ = maximum axial force in the column due to earthquake and gravity loads. P_{EQ} is computed by the modal analysis using IARS. M is the design moment defined as the maximum value of M_{EQ} or M_{GR}, where M_{EQ} and M_{GR} are the average of the two column end moments due to earthquake and gravity loads. Since redesign of the members to provide the required capacities results in changes of member stiffnesses, the procedure is necessarily iterative. However, in the example which follows, the capacities were changed from the preliminary design, but the slight changes in stiffnesses were ignored. Moreover, load factors were not applied to the gravity and earthquake loads in the design of members.

EXAMPLE OF A 10-STORY STEEL FRAME

The steel moment-resisting frame used in this study was initially designed according to the 1973 Uniform Building Code. Elevation and member sizes of the frame are shown in Fig. 3. The frame is assumed to have an adequate bracing system to resist out-of-plane motion. Based on the preliminary member sizes, the frame is redesigned by the inelastic design procedure as described previously. Assuming a design ductility ratio of 4, a 5% damping, and a peak ground acceleration of 0.33 g, the proposed IARS as shown in Fig. 2 scaled to 1/3 of the original values is used to compute the required member strength. To evaluate the adequacy of the design, the inelastic responses of the frame are then calculated by dynamic time-history analysis using the computer program FRIEDA. In performing the analysis, shear deformation, axial deformation in the girders, soil-structure interaction, and the P-Δ effect are neglected. In measuring the response, the rotational ductility ratio (μ_θ) is used for columns, whereas the moment ductility ratio (μ_M) is used for girders. For detailed discussions of the relative merits of these two ductility ratios, the reader is referred to Lai (1).

Five previously generated artificial motions scaled to 0.33 g peak acceleration with duration equal to 20 secs are used in the study. Fig. 4 shows the mean and mean ± one standard deviation of the maximum ductility ratios for columns and girders. The mean ductility ratio is the average of five "maximum local ductility ratios" resulting from time-history analyses for the set of motions. As shown in Fig. 4, the ductility ratios for upper-floor exterior columns are generally greater than the design ductility of 4. For interior columns, the ductility ratios are quite compatible with the design value for all stories. In exterior and interior girders, the ratios in the lower stories are somewhat larger than the design value, while in the upper floors the ratios are much smaller. This is not surprising, since the moment capacities of these upper-floor girders are controlled by the gravity requirement, $w\ell^2/8$. Thus the member capacities are larger than required to resist the earthquake, and smaller local ductility ratios resulted.

In summary, the design method based on the proposed inelastic-response spectra leads to a generally reliable design but is slightly unconservative for upper-story exterior columns.

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REFERENCES

1. Lai, S.P., "On Inelastic Response Spectra for Aseismic Design," Report R78-18, Dept. of Civil Engrg., M.I.T., July 1978.

2. Newmark, N.M. and Hall, W.J., "Procedures and Criteria for Earthquake Resistant Design," Building Practices for Disaster Mitigation, National Bureau of Standards, Feb., 1973, pp. 209-236.

3. Riddell, R. and Newmark, N.M., "Statistical Analysis of the Response of Nonlinear Systems Subjected to Earthquakes," Civ. Engrg. Studies, Structural Res. Series No. 468, Univ. of Illinos, Urbana-Champaign, Aug. 1979.

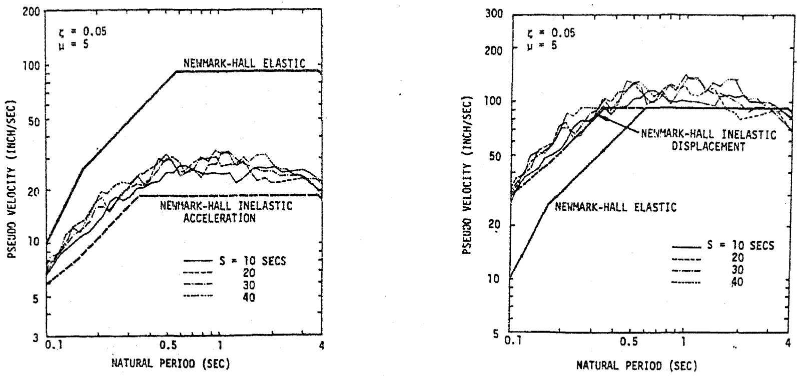

Fig. 1 - Mean IARS and IDRS for Different Strong-Motion Durations

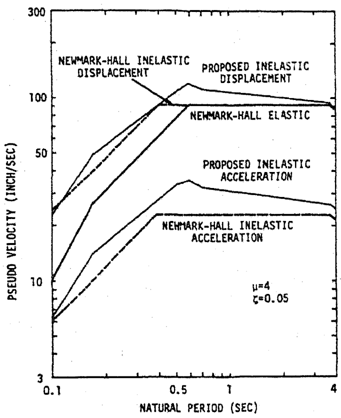

Fig. 2 - Proposed Inelastic Response Spectrum

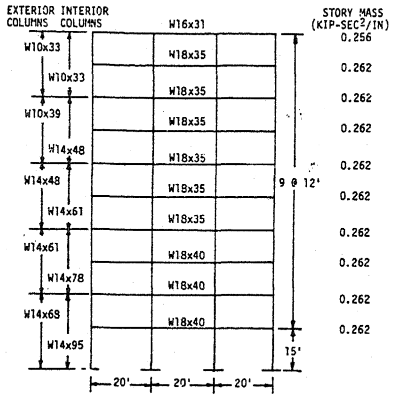

Fig. 3 - 10-Story Steel Building Elevation

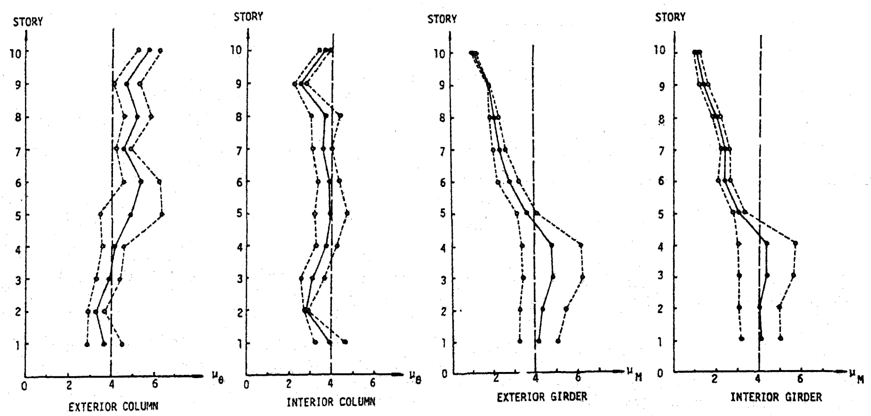

Fig. 4 - Maximum Column and Girder Ductility Ratios for the 10-Story Building